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Technical Report CERC-93-14
August 1993

Revetment Stability Tests for Sargent Beach, Texas

by Robert D. Carver, Willie J. Dubose,
John M. Heggins, Brenda J. Wright
Coastal Engineering Research Center

WES

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Preface

The model investigation described herein was requested by the U.S. Army Engineer District, Galveston (SWG), in a letter to the U.S. Army Engineer Waterways Experiment Station (WES) in June 1991. Funding authorization was granted by SWG in July 1991. Tests were conducted intermittently between July 1991 and July 1992.

The study was conducted by personnel of the WES Coastal Engineering Research Center (CERC) under the general direction of Dr. James R. Houston, Director, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Director, CERC. Direct guidance was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division, and D. Donald Davidson, Chief, Wave Research Branch (WBR). Tests were conducted by Ms. Brenda J. Wright and Messrs. W. G. Dubose and J. M. Heggins, WRB, under the direction of Mr. R. D. Carver, Principal Investigator, WRB. This report was prepared by Ms. Wright and Messrs. Carver, Dubose, and Heggins. The Galveston District Project Manager was Captain R. Schultz.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic meters per second
feet	0.3048	meters
inches	2.54	centimeters
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
tons (2,000 lb, mass)	907.1847	kilograms

1 Introduction

The Prototype

Sargent Beach, Texas, is located on the Gulf coast about 60 miles[1] southwest of Galveston, TX (Figure 1). The beach, which provides protection for the Gulf Intercoastal Waterway (GIWW), is experiencing an average erosion rate of 30 ft per year. Left unchecked, the beach will be breached in a few years and the GIWW will be directly exposed to Gulf waves. A revetment, approximately 8 miles long, has been proposed to protect the eroding shoreline.

Purpose of Model Investigation

The objective of this study was to investigate, via a two-dimensional coastal model, alternate designs for the proposed revetment. The first two designs investigated were similar except that one plan used 5-ton stone armor, whereas the other was protected by 6.2-ton concrete blocks. When the stability of the first concrete block design proved to be marginal, block shape and, thus, gross porosity of the armor layer were modified in an effort to achieve satisfactory stability. Finally, the shape of the concrete blocks was optimized and wave overtopping rates were determined for the recommended section.

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vi.

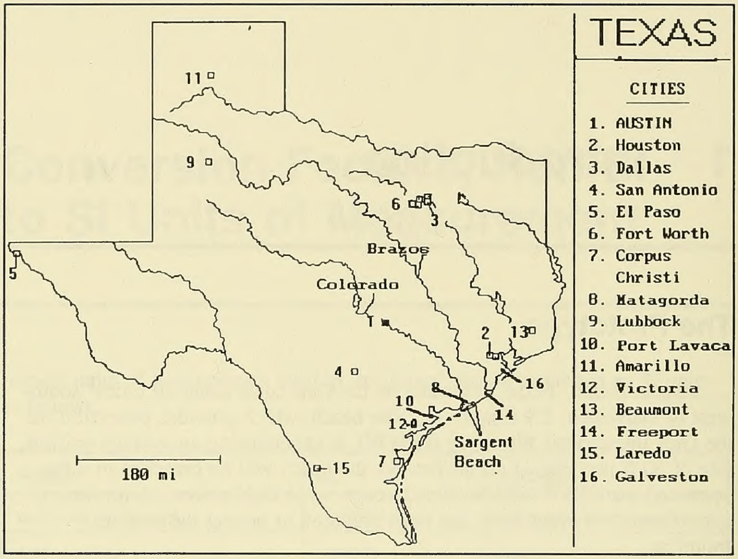

Figure 1. Location map

2 The Model

Model-Prototype Scale Relationships

Tests were conducted at a geometrically undistorted scale of 1:24, model to prototype. Scale selection was based on the sizes of model armor available compared with the estimated size of prototype armor required for stability, preclusion of stability scale effects (Hudson 1975), and capabilities of the available wave tank. Based on Froude's model law (Stevens 1942) and a linear scale of 1:24, the following model-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristic	Dimension	Model-Prototype Scale Relation
Length	L	$L_r = 1:24$
Area	L^2	$A_r = L_r^2 = 1:576$
Volume	L^3	$V_r = L_r^3 = 1:13824$
Time	T	$T_r = L_r^{1/2}$ 1:4.90

The specific weight of water used in the model was assumed to be 62.4 pcf, compared to that of seawater, 64.0 pcf; also, specific weights of model revetment construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3$$

W_a = weight of an individual armor unit, pounds

m, p = model and prototype quantities, respectively

γ_a = specific weight of an individual armor unit, pounds per cubic foot

L_m/Lp = linear scale of the model

S_a = specific gravity of an individual armor unit relative to the water in which it was placed, i.e., $S_a = \gamma_a/\gamma_w$

Test Equipment and Facilities

All tests were conducted in a concrete wave flume 3 ft wide and 150 ft long (Figure 2). A 1V on 100H slope, representative of the existing prototype sea bottom, was molded Gulfward of the test section. Irregular waves were generated by a hydraulically actuated piston-type wave machine. The test section was installed approximately 84 ft from the wave board.

Wave data were collected on electrical capacitance wave gauges. Wave signal generation and data acquisition were controlled using a DEC MicroVax I computer. Wave data analysis was accomplished using a DEC VAX 3600.

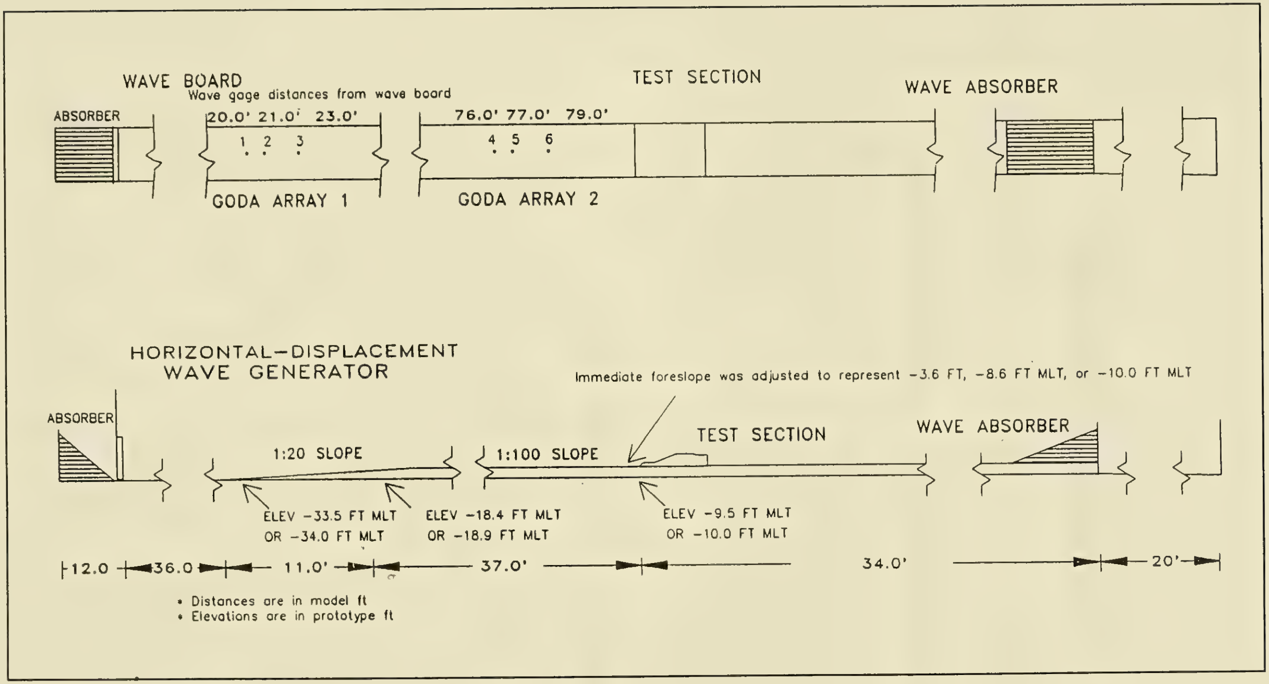

Figure 2. Concrete wave flume

3 Tests and Results

Method of Constructing Test Sections

All experimental revetment sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale revetments. Underlayer stone was added by shovel and smoothed to grade by hand or with trowels. Armor units used in the cover layer were specially placed with their least dimension (2.5 ft) perpendicular to the underlayer. After each test, the armor units were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced.

Selection of Test Conditions

Based on siting of the breakwater in shallow water, tests were conducted with a TMA spectrum using peak periods (T_p) of 8 and 10 sec. The wave basin was calibrated for still-water levels (swl's) of +4, +7, +9.5, and +14 ft mean low tide (mlt) for assumed erosion depths of -3.6 and -8.6 ft mlt (Figure 3). Thus, as summarized below, eight testing depths were considered.

Erosion Depth, ft, mlt	swl, ft, mlt	Total Depth at Toe, ft
-3.6	+4.0	7.6
-3.6	+7.0	10.6
-3.6	+9.5	13.1
-3.6	+14.0	17.6
-8.6	+4.0	12.6
-8.6	+7.0	15.6
-8.6	+9.5	18.1
-8.6	+14.0	22.6

Prior to testing of the final plans, it was decided that an erosion depth of -10.0 ft mlt was plausible. Using this new assumption and adding a +11.5-ft swl yielded the following five depths:

Erosion Depth, ft, mlt	swl, ft, mlt	Total Depth at Toe, ft
-10.0	+4.0	14.0
-10.0	+7.0	17.0
-10.0	+9.5	19.5
-10.0	+11.5	21.5
-10.0	+14.0	24.0

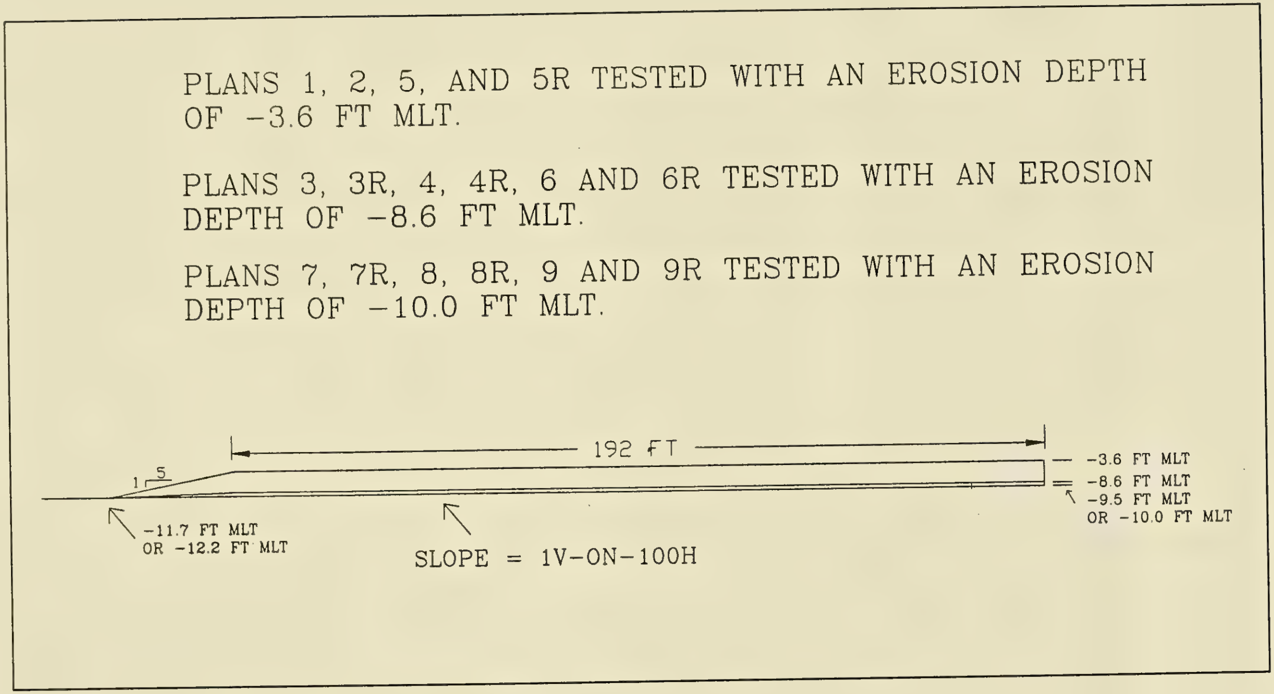

Figure 3. Summary of assumed erosion conditions

Wave heights were measured about 100 ft (prototype) in front of the test sections. Goda and Suzuki's (1976) method was used to resolve the incident and reflected spectra.

Descriptions and Test Results for the Stone-Armored Plans

Plan 1 (Figure 4 and Photos 1 and 2) was constructed to a crown elevation of +7-ft mlt and used an armor slope of 1V on 2.5H. A crown width of 20 ft, equivalent to four armor-stone diameters, was used. The 5-ton armor stones (specific weight ≈ 165 pcf), which are approximately 2.5-ft thick, 4-ft wide, and 6-ft long, were specially placed with the 2.5-ft dimension perpendicular to the slope. The majority of the stones were also placed with their long axis perpendicular to the wave crest (upslope). It was assumed that this orientation would minimize uplift forces.

Plan 1 was subjected to the 24-step test delineated in Table 1. Some landward displacement from the shore-side crest of the structure of the 200- to 1,000-lb stone was observed at the +4-ft swl. Minor reorientation of some 200- to 1,000-lb toe stone also was observed. No armor stone movement was detected. The +7-, +9.5-, and +14-ft swl's supported progressively larger waves; however, much of this energy passed over the structure and only two additional 200- to 1,000-lb stones shore-side of the erest were displaced. Photos 3-5 show the after-testing condition of the structure.

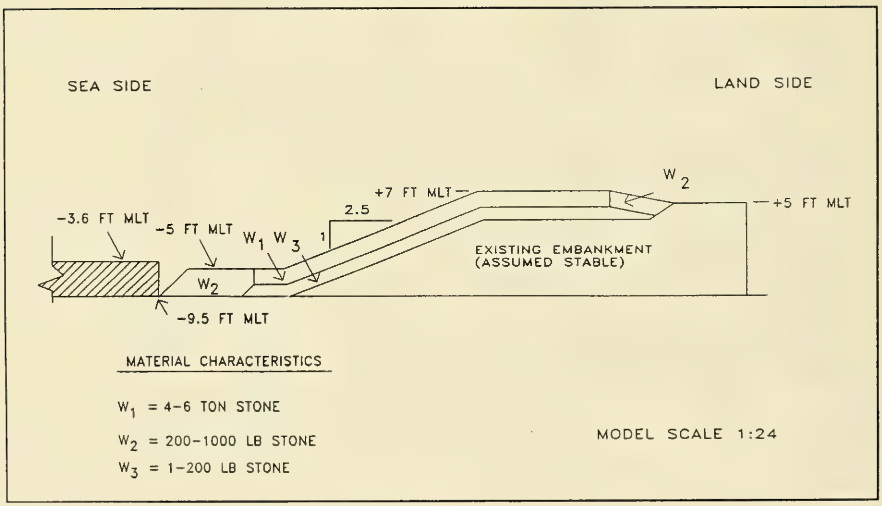

Figure 4. Stone revetment cross section, Plan 1

Plan 2 (Figure 5) was identical to Plan 1, except the upper toe elevation was raised from -5.0 ft to -3.6 ft mlt. This plan represented a repair or addition of material that might be made to the existing prototype structure. Therefore, the remaining portions of the structure were not rebuilt.

Plan 2 was tested for the six-step storm given in Table 2. Only the +4-ft swl was tested, since this water level appeared to be the most critical to toe stability in previous tests of Plan 1. Plan 2 proved to be stable. Similar to Plan 1, minor reorientation of some 200- to 1,000-lb toe stone was observed. Also, several additional 200- to 1,000-lb stones were displaced from the shoreside crest of the structure. The after-testing condition of the structure is shown in Photos 6-8.

Plan 3 (Figure 6) was identical to Plan 1 except it was assumed that another 5 ft of erosion occurred seaward of the section. Thus, the effective water depth was increased by 5 ft at each swl.

As anticipated, some additional reorientation of the 200- to 1,000-lb toe stone was observed. The most significant movement observed during testing of Plan 3 occurred along the crest at the +7.0-ft swl (steps 7-12 of the hydrograph given in Table 3) and consisted of minor shoreward movement of several 4- to 6-ton stones. This movement, shown in Photos 9 and 10, was not extensive enough to jeopardize the integrity of the section.

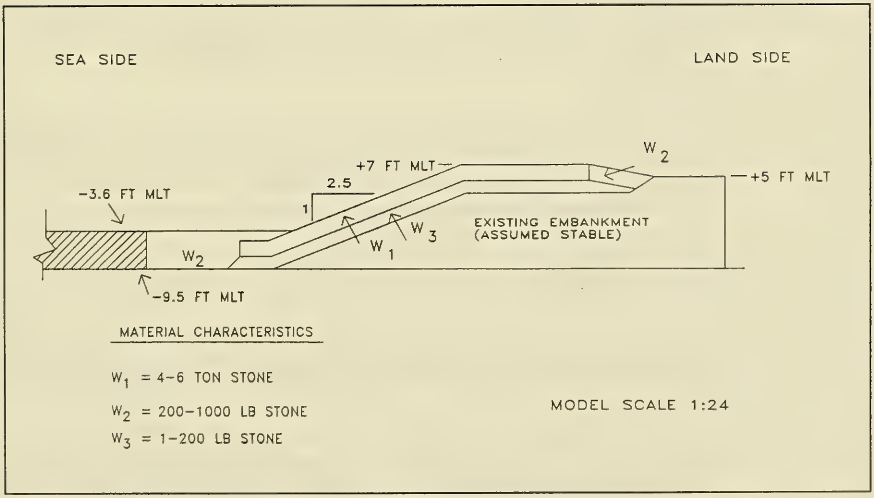

Figure 5. Stone revetment cross section, Plan 2

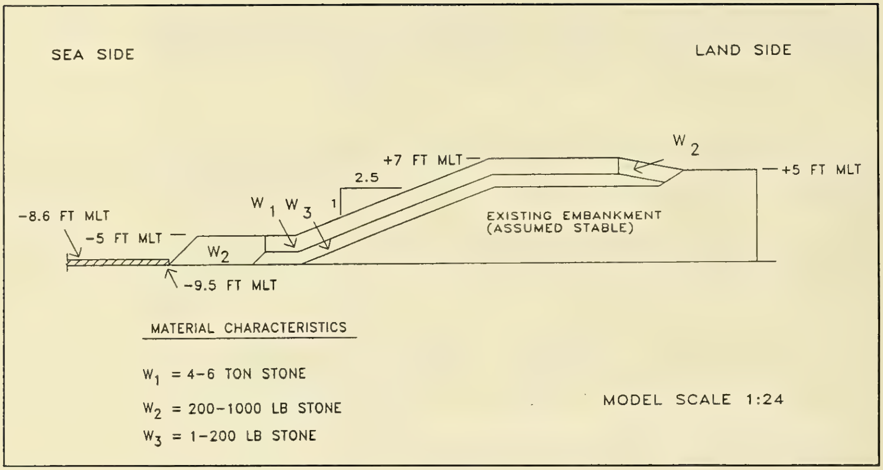

Figure 6. Stone revetment cross section, Plans 3 and 3R

Plan 3R (Figure 6) was the same as Plan 3 except the model structure was rebuilt and the majority of the armor stones were placed with their long axis parallel to the incoming wave crest. This plan thus served to verify the response of the 200- to 1,000-lb toe and shore-side crest stone and compare the stability of the 4- to 6-ton armor for the two possible long axis orientations.

Plan 3R was tested for the abbreviated worst case eight-step storm given in Table 4. Similar to previous plans, some reorientation of the 200- to 1,000-lb toe stone was observed and several 200- to 1,000-lb stones were displaced from the shore-side crest of the structure. The 5-ton armor was generally stable; however, one stone was displaced from the crest of the revetment during step 8 (Table 4 at the -8.6-ft mlt toe condition). The after-testing condition of the structure is shown in Photos 11 and 12.

Summary of Results for the Stone-Armored Plans

Results of tests conducted with stone armor (Plans 1, 2, 3, 3R) show the 4- to 6-ton armor stone to be stable for any reasonable combination of swl, wave period, and wave height that can be expected to occur. The 200- to 1,000-lb toe and berm stone is only minimally adequate; therefore, it is recommended that the weight of this stone be increased.

Descriptions and Test Results for the Concrete Block Plans

Plan 4 (Figure 7) used the same overall geometry (crown elevation, crown width, and armor slopes) as previous plans tested at the -8.6-ft erosion depth. However, the 5-ton armor stone was replaced by 6.2-ton concrete blocks (specific weight = 150 pcf). The 2.5-ft-thick, 5.5-ft-wide, and 6-ft-long blocks were uniformly placed with their least dimension perpendicular to the slope. Also, in an effort to reduce toe and shore-side crest stone movement, the 200- to 1,000-lb stones were replaced by 200- to 2,000-lb material. Plan 4 was tested with the same wave conditions used in tests of Plan 3 (Table 3).

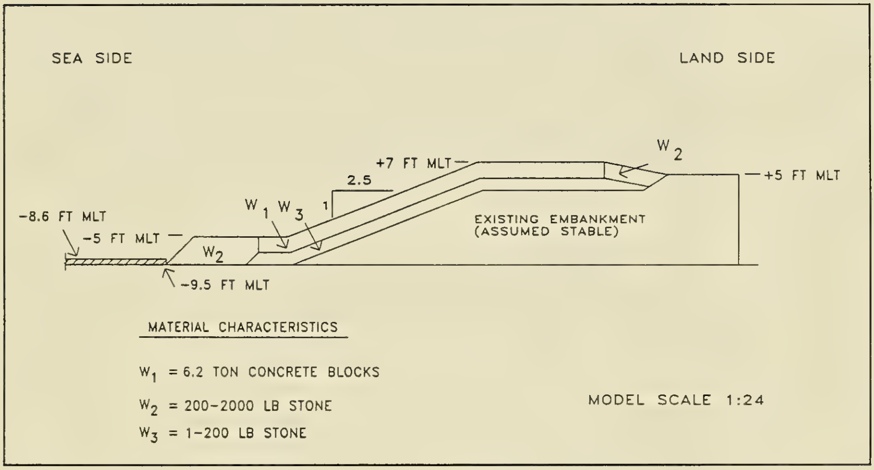

Figure 7. Concrete block revetment cross section, Plans 4 and 4R

Seaward slippage of the 6.2-ton toe blocks was initiated at the +4.0-ft swl. Also, as the toe blocks moved slightly seaward, the next five rows of blocks above them packed downslope. This left a gap of about 6-9 in. (prototype) between the upper row of slope blocks and the first row of crest blocks. Several of the seaside blocks were observed to lift slightly during wave attack; however, none were displaced. A small amount of additional slippage of the seaward blocks was observed during the +7.0- and +9.5-ft swl's (the above-described gap was about 10 in. wide by conclusion of the +9.5-ft swl). Continued lifting and reseating of the blocks was observed at the +14-ft swl, with one block being displaced downslope during step 24 (Table 3 at the -8.6-ft mlt toe condition). Photos 13 and 14 show the section after wave attack. The gap between the upper row of slope blocks and the first row of crest blocks was about 1 ft (prototype) at the conclusion of the test. The stability response of the concrete blocks in Plan 4 is considered marginally acceptable.

The 200- to 2,000-lb toe and crest stone showed significantly improved stability relative to the 200- to 1,000-lb stone used in previous plans. As would be expected, a few of the smaller stones still were displaced.

Plan 4R (Figure 7) was a rebuild of Plan 4. It was tested using the 16-step storm (Table 5) at the -8.6-ft mlt toe condition to verify the stability response of the concrete blocks.

Test results at the +4-ft swl verified the movement observed during tests of the previous plan, i.e., there was a slight seaward slippage of the toe blocks, which allowed the next five rows of blocks above them to pack downslope. A small amount of additional slippage of the seaward blocks was observed during the +7.0- and +9.5-ft swl's. The +14-ft swl displaced four blocks downslope during step 16 (Table 5 at the -8.6-ft mlt toe condition (Photos 15 and 16)). A comparison of Photos 13 and 14 with 15 and 16 shows the repeat test produced results similar to the original; however, three more seaward blocks were displaced. The structure did not fail; however, any displacement is cause for concern with a one-layer armor system. Therefore, as with Plan 4, stability of the concrete blocks is rated only marginally acceptable. The 200- to 2,000-lb shore-side crest and toe stone performed similar to Plan 4; i.e., a few smaller stones were displaced, but the overall stability of this material was good.

Plan 5 (Figure 8) was similar to Plan 4 except it was assumed that the erosion depth in front of the structure was 5 ft less (sea bottom was raised from -8.6 ft mlt to -3.6 ft mlt). Also, the upper toe elevation was raised from -5.0 ft mlt to -3.6 ft mlt. Plan 5 was subjected to the same 24-step test as Plan 1 (Table 1). Testing at the +4-ft swl produced no movement of the concrete blocks. A few of the smaller 200- to 2,000-lb shore-side crest stones were displaced. Two additional crest stones were displaced during continued testing at the +7-, +9.5-, and +14-ft swls. No movement of the concrete blocks could be detected at any of the swl's. Photos 17 and 18 show the structure after wave attack.

Plan 5R (Figure 8) was a rebuild of Plan 5. It was tested to verify the stability response of the concrete blocks for the -3.6-ft erosion depth. Plan 5R was subjected to the 16-step test listed in Table 6. Results verified the outcome of the initial test, i. e., no movement of the concrete blocks was detected at any of the swl's until the final step of the test was reached. The 10-sec, 13.2-ft waves at the +14-ft swl (step 16, Table 6) displaced one block from the seaward face (Photos 19-21). Also, a few of the smaller 200- to 2,000-lb shore-side crest stones were displaced at the +4- and +7-ft swl's.

Plan 6 (Figure 9 and Photos 22-24) was similar to Plans 4, 4R, 5, and 5R, except armoring was provided by 6.0-ton concrete blocks, 3.0-ft thick, 5.25-ft wide, and 5.25-ft long (Figure 10). The blocks were uniformly placed with their least dimension perpendicular to the slope. A 1.0-ft-long, 0.25-ft-wide and 3.0-ft-deep indentation was formed in each side of each block (Figure 10). The indentations served to increase the porosity of the block cover layer and reduce the buildup of pressures beneath the blocks. It was also felt that the

indentations might be advantageous in the lifting and placing of blocks during construction.

Testing of Plans 1-6 showed wave conditions for the -8.6-ft mlt erosion depth to be significantly more severe than those observed at the -3.6-ft depth; therefore, stability tests for Plans 6 and 6R were conducted only at the -8.6-ft depth and it was assumed results could be conservatively applied to any lesser depth. Plan 6 was subjected to the 16-step test given in Table 5. The concrete blocks proved to be stable for all wave conditions; however, a few of the smaller 200- to 2,000-lb shore-side crest stones were displaced (Photos 25-27).

Plan 6R (Figure 9) was a rebuild of Plan 6. It was tested to verify the stability response of the modified concrete blocks. Subjection to the abbreviated worst case hydrograph given in Table 4 produced the same results as the initial test, i.e., no movement of the concrete blocks was detected for any of the wave conditions. Photos 28-30 show the final condition of the test section.

The concrete blocks used on Plans 6 and 6R appeared to be conservatively stable. Therefore, in an effort to reduce cost without sacrificing stability, the blocks were redesigned with a 2.5-ft thickness and a slightly increased porosity (approximately 4 percent).

Plan 7 (Figure 11) was armored with the new 6.0-ton concrete blocks. The 2.5-ft-thick, 5.75-ft-wide, and 5.75-ft-long blocks (Figure 12) were uniformly placed with their least dimension perpendicular to the slope. The toe and splash apron were protected by 200- to 2,000-lb stone.

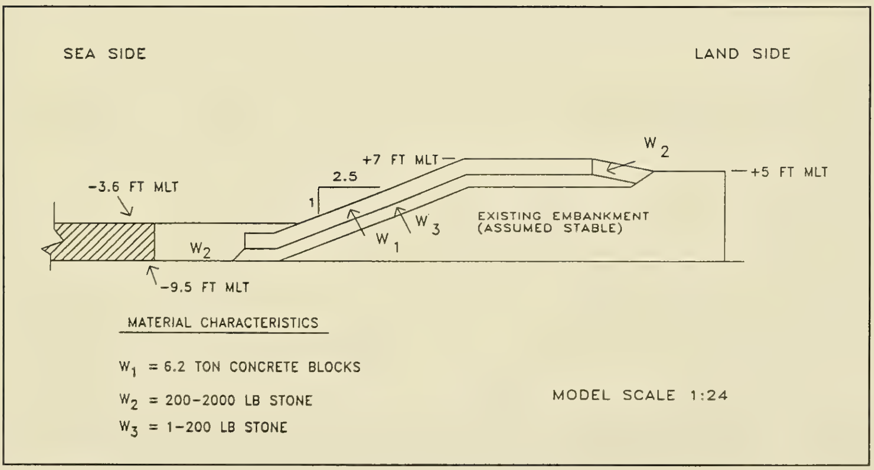

Figure 8. Concrete block revetment cross section, Plans 5 and 5R

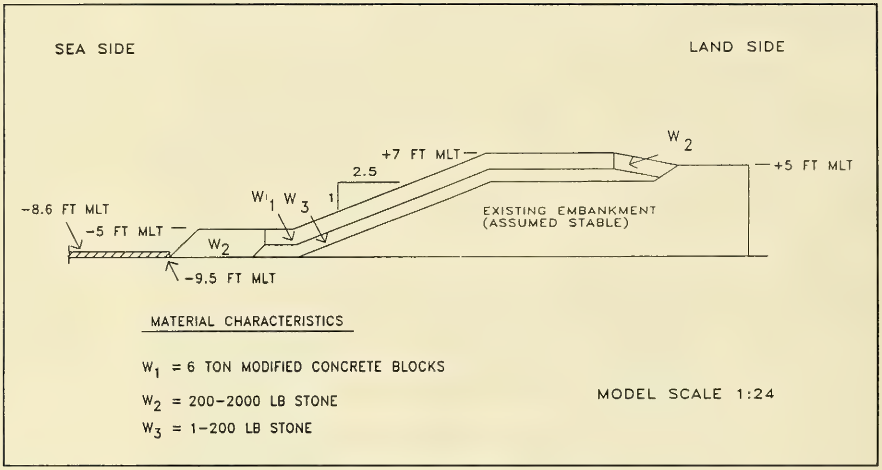

Figure 9. Modified concrete block revetment cross section, Plans 6 and 6R

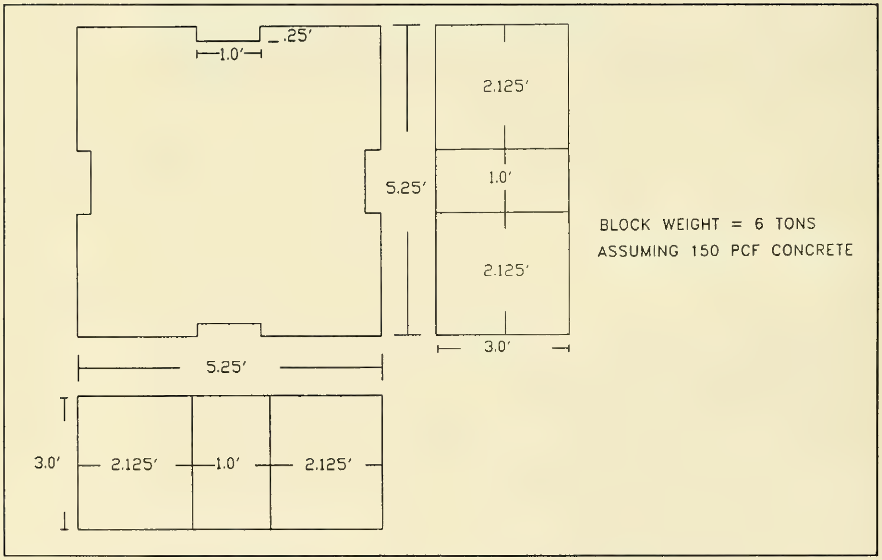

Figure 10. Details of modified concrete block used on Plans 6 and 6R

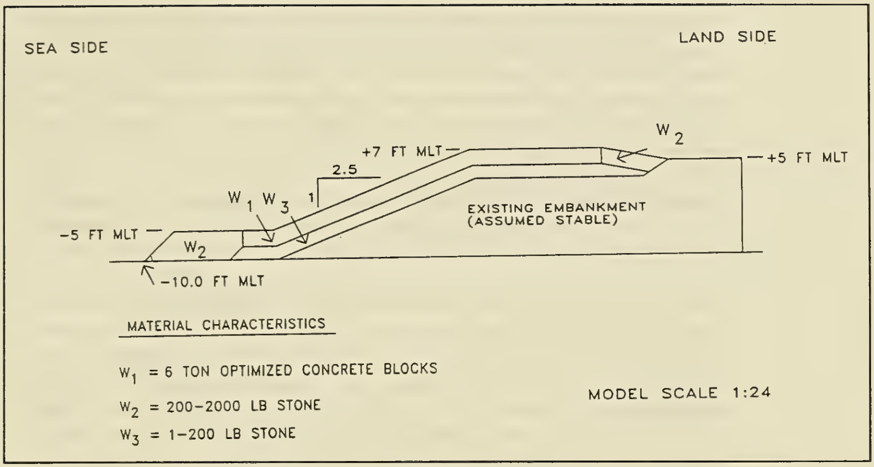

Figure 11. Optimized concrete block revetment cross section, Plans 7 and 7R

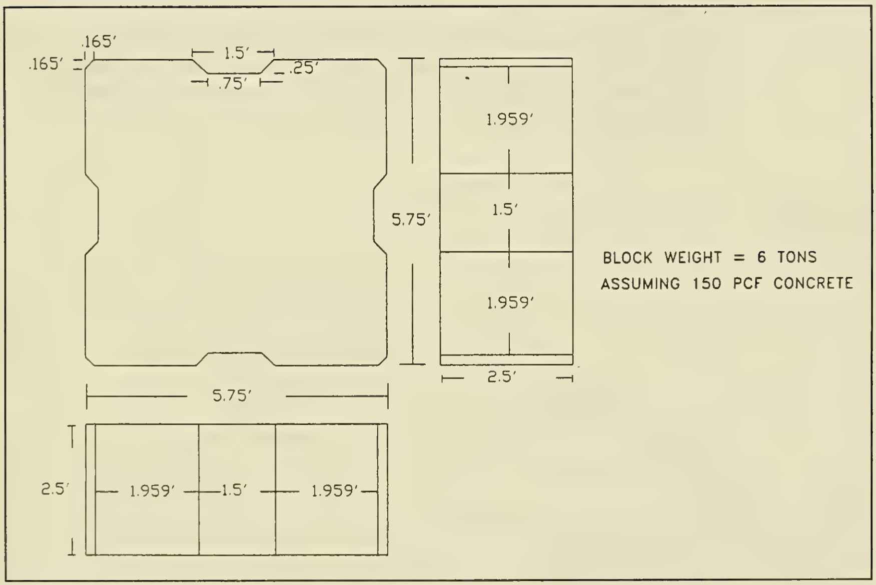

Figure 12. Details of optimized concrete block used on Plans 7, 7R, 8, 8R, 9, and 9R

Plan 7 was subjected to the 20-step test given in Table 7. Slight seaward shifting of the first row of concrete blocks was initiated at the +4-ft swl. As the toe blocks shifted seaward, the six rows of onslope blocks packed downslope, leaving a gap between them and the three rows of crest blocks. This slow progressive movement continued through the +7-ft swl. At the conclusion of testing, the gap between the slope and crest blocks varied from a few inches to about 1.5 ft (prototype). Individual concrete blocks appeared to be hydraulically stable, i.e., no lifting or displacement was observed. Photos 31-33 show the overall condition of the section after wave attack and Photo 34 shows details of the block separation.

Plan 7R (Figure 11) was a rebuild of Plan 7. Subjection to the 10-step hydrograph given in Table 7 verified results of the first test, i.e., a gap, varying in width from a few inches to 1.5 ft, developed between the slope and crest blocks as a result of toe slippage and subsequent downslope slippage of the slope blocks. Again, no lifting or displacement of individual blocks was observed. Photos 35-38 show the final condition of the test section. As can be observed in after-testing photos, the 200- to 2,000-lb toe and crest stone experienced some displacement.

Plan 8 (Figure 13 and Photos 39 and 40) was the same as Plan 7, except that the toe stone weight was increased to 200 to 4,000 lb in an effort to improve its stability and hopefully reduce sliding of the concrete blocks. Subjection to the same wave conditions as Plan 7 produced improved results; i.e., the toe stone was stable and the maximum gap between the slope and crest blocks was reduced to about 1 ft (prototype). Photos 41-44 show the structure after wave attack.

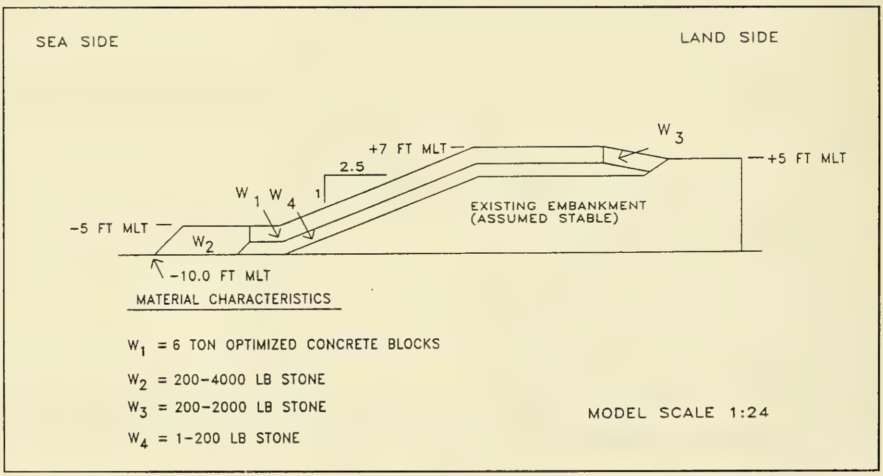

Figure 13. Optimized concrete block revetment cross section, Plans 8 and 8R

Plan 8R (Figure 13) was a rebuild of Plan 8. Subjection to the hydrograph given in Table 8 verified results of the first test, i.e., a gap developed between the slope and crest blocks as a result of toe slippage and subsequent downslope slippage of the slope blocks. Again, no lifting or displacement of individual blocks was observed. Photos 45 and 46 show the final condition of the test section.

Plan 9 (Figure 14) was similar to Plan 8, except that an additional row of concrete blocks was added to the toe and a proportionate amount of 200- to 4,000-lb toe stone was removed. Testing with the same wave conditions as Plan 8 produced similar results, i.e., a gap varying from a few inches to 1 ft developed between the slope and crest blocks as a result of toe slippage. As shown in Photos 47-49, the final condition of the structure was similar to that observed for Plans 8 and 8R.

Plan 9R (Figure 14) was a rebuild of Plan 9. It was tested with the 10-step hydrograph given in Table 2. Again, a gap developed between the slope and crest blocks as a result of toe slippage and subsequent downslope slippage of the slope blocks. Final condition of the structure (Photos 50-52) was similar to that observed for Plans 8, 8R, and 9, i.e., a gap that varied from a few inches to about 1 ft developed between the slope and crest blocks.

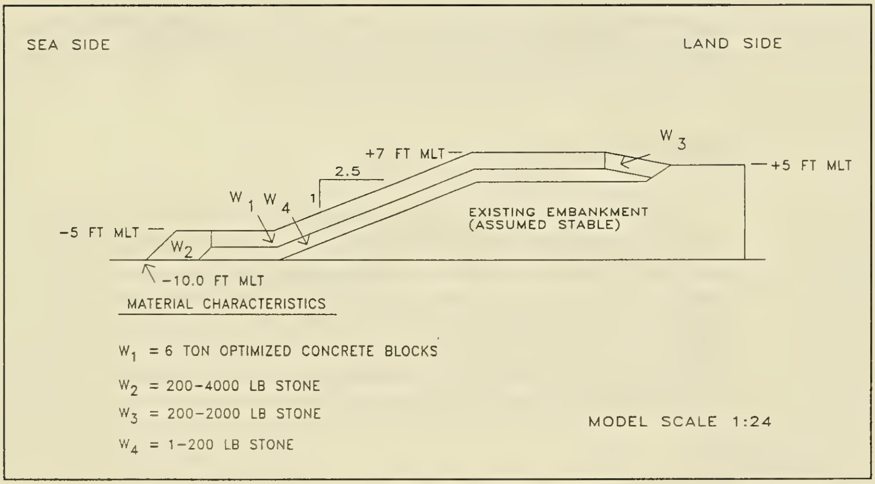

Figure 14. Optimized concrete block revetment cross section, Plans 9 and 9R

Summary of Results for the Concrete Block Plans

As evidenced in tests of Plans 4 and 4R, stability of the original 2.5-ft by 5.5-ft by 6-ft concrete blocks is only marginally acceptable for the -8.6-ft erosion depth. However, tests of Plans 5 and 5R show the blocks to be acceptable for the 3.6-ft erosion depth. The 3-ft by 5.25-ft by 5.25-ft modified concrete blocks (Plans 6 and 6R) are completely stable for the -8.6-ft erosion depth and can be assumed stable for any lesser depth. Tests at the -10-ft erosion depth (Plans 7-9R), show the 2.5-ft by 5.75-ft by 5.75-ft block design (Figure 12) to be hydraulically stable; however, a gap may develop between the slope and crest blocks as a result of toe slippage and subsequent downslope slippage of the slope blocks.

The 200- to 2,000-lb toe and crest stone used on Plans 4, 4R, 5, 5R, 6, and 6R showed significantly improved stability relative to the 200- to 1,000-lb stone used on the stone armor plans. Based on an estimate of the material volumes in the as-built model sections and the number of stones moved during wave attack, displacement of 1 to 2 percent of the toe stone and 2 to 4 percent of the crest stone can be expected during wave attack at the +4- and +7-ft swl's.

Initial tests at the -10-ft erosion depth (Plans 7 and 7R) showed the 200- to 2,000-lb stone to be marginal for the toe of the structure. Therefore, this weight was increased to 200 to 4,000 lb for Plans 8-9R.

Wave Overtopping Tests

Limited wave overtopping tests were conducted on Plan 8. To obtain model overtopping rates, calibrated containers were placed behind and above the model revetment to collect water overtopping the structure (water was transferred by pump to the overhead container). Water surface elevations in the overtopping containers were measured with a point gauge before and after each test to determine the total quantity of overtopping. Photo 53 shows a general view of the model setup. Overtopping tests were conducted at swl's of +2.5 and +4.0 ft mlt for the following wave conditions:

SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft	Overtopping Rate Q, cfs/ft
+2.5	5.0	5.5	0.05
+2.5	6.0	4.5	0.02
+2.5	6.0	6.0	0.16
+2.5	7.0	5.5	0.19
+2.5	7.0	7.0	0.35
+4.0	8.0	4.0	0.61
+4.0	8.0	6.0	1.20
+4.0	8.0	7.8	2.41
+4.0	8.0	9.9	3.15
+4.0	10.0	4.1	0.86
+4.0	10.0	6.2	1.46
+4.0	10.0	8.0	3.13
+4.0	10.0	10.4	3.16

Overtopping rate is presented as a function of incident wave height and wave period for the +4-ft swl in Figure 15. These data show that both wave periods give similar results: wave heights of 4 ft and above produce major overtopping (rates in excess of 0.5 cfs/ft), and maximum overtopping rates of approximately 3 cfs/ft can be expected for the largest waves that reach the structure.

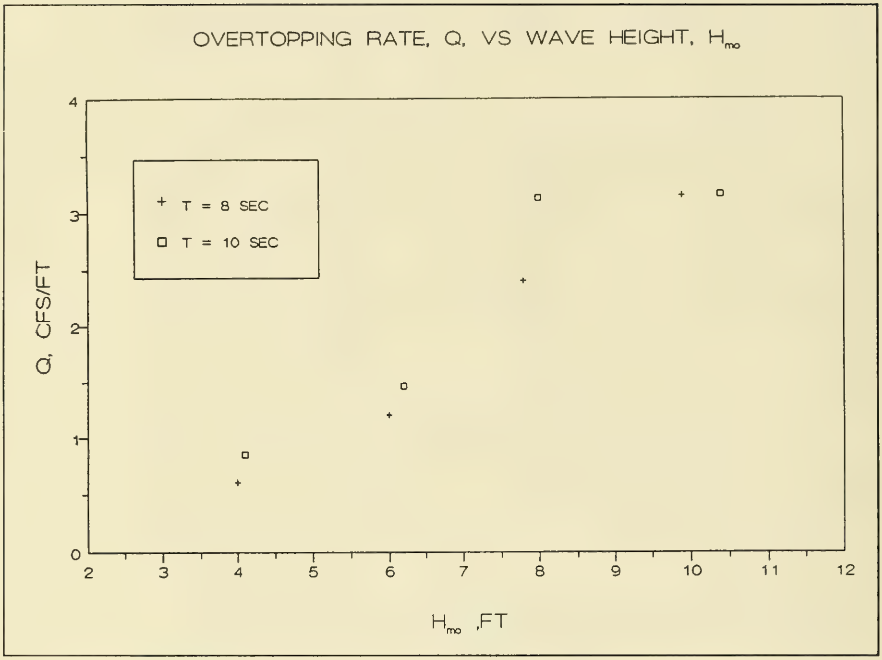

Figure 15. Overtopping rate for +4-ft swl

4 Conclusions

Based on assumptions, tests, and results reported herein, it is concluded that:

a. The 4- to 6-ton armor stone used in Plans 1, 2, 3, and 3R is stable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at swl's of +4.0- to +14.0 ft mlt with assumed scour depths of -3.6 and -8.6 ft mlt. The 200- to 1,000-lb toe and crest stone is only minimally adequate; therefore, it is recommended that the weight of this stone be increased to the 200- to 2,000-lb range.

b. The 6.2-ton concrete blocks used in Plans 4 and 4R are only marginally acceptable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at swl's of +4.0- to +14.0-ft mlt with an assumed scour depth of -8.6 ft mlt. The 200- to 2,000-lb toe and crest stone showed significantly improved stability relative to the 200- to 1,000-lb stone used in Plans 1, 2, 3, and 3R.

c. The 6.2-ton concrete blocks used in Plans 5 and 5R are acceptable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at swl's of +4.0 to +14.0 ft mlt with an assumed scour depth of -3.6 ft mlt. Again, the 200- to 2,000-lb toe and crest stone showed significantly improved stability relative to the 200- to 1,000-lb stone used in Plans 1, 2, 3, and 3R.

d. The modified concrete blocks used in Plans 6 and 6R are stable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at swl's of +4.0- to +14.0-ft mlt with an assumed scour depth of -8.6 ft mlt. As would be expected, the 200- to 2,000-lb toe and crest stone proved to be stable with the modified concrete blocks. It is reasonable to assume that the modified concrete blocks would prove stable for any lesser erosion depth.

e. The 6-ton concrete blocks used in Plans 7, 7R, 8, 8R, 9, and 9R are stable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at swl's of +4.0 to +14.0 ft mlt with an assumed

scour depth of -10 ft mlt. The 200- to 2,000-lb toe stone experienced a significant increase in movement at the -10-ft depth; therefore, the 200- to 4,000-lb stone tested on Plans 8, 8R, 9, and 9R is recommended for prototype use.

f. Plan 8 appears to be the best plan in terms of stability relative to material sizes/amounts.

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Goda, Y. and Suzuki, Y. (1976). "Estimation of incident and reflected waves in random wave experiments." *Proceedings, 15th International Conference on Coastal Engineering*. Honolulu, HI.

Hudson, R. Y. (1975). "Reliability of rubble-mound breakwater stability models," Miscellaneous Paper H-75-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Stevens, J. C. (1942). "Hydraulic models," *Manuals of Engineering Practice No. 25*, American Society of Civil Engineers, New York.

Table 1
Summary of Test Conditions; Plans 1 and 5
Erosion Depth = -3.6 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	5.3
2	+4.0	8.0	5.8
3	+4.0	8.0	6.3
4	+4.0	10.0	5.4
5	+4.0	10.0	6.3
6	+4.0	10.0	7.0
7	+7.0	8.0	6.1
8	+7.0	8.0	8.1
9	+7.0	8.0	8.9
10	+7.0	10.0	7.1
11	+7.0	10.0	8.7
12	+7.0	10.0	9.6
13	+9.5	8.0	7.2
14	+9.5	8.0	9.3
15	+9.5	8.0	10.1
16	+9.5	10.0	7.3
17	+9.5	10.0	9.4
18	+9.5	10.0	10.7
19	+14.0	8.0	7.8
20	+14.0	8.0	10.8
21	+14.0	8.0	12.2
22	+14.0	10.0	8.3
23	+14.0	10.0	11.9
24	+14.0	10.0	13.2

Table 2
Summary of Test Conditions; Plan 2
Erosion Depth = -3.6 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	5.3
2	+4.0	8.0	5.8
3	+4.0	8.0	6.3
4	+4.0	10.0	5.4
5	+4.0	10.0	6.3
6	+4.0	10.0	7.0

Table 3
Summary of Test Conditions; Plans 3 and 4
Erosion Depth = -8.6 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	5.9
2	+4.0	8.0	7.3
3	+4.0	8.0	8.1
4	+4.0	10.0	5.9
5	+4.0	10.0	7.6
6	+4.0	10.0	8.1
7	+7.0	8.0	6.4
8	+7.0	8.0	8.5
9	+7.0	8.0	9.5
10	+7.0	10.0	6.8
11	+7.0	10.0	8.6
12	+7.0	10.0	9.6
13	+9.5	8.0	7.3
14	+9.5	8.0	9.6
15	+9.5	8.0	10.6
16	+9.5	10.0	7.7
17	+9.5	10.0	10.1
18	+9.5	10.0	10.9
19	+14.0	8.0	8.0
20	+14.0	8.0	11.0
21	+14.0	8.0	12.6
22	+14.0	10.0	8.5
23	+14.0	10.0	12.5
24	+14.0	10.0	14.0

Table 4
Summary of Test Conditions; Plans 3R and 6R
Erosion Depth = -8.6 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	8.1
2	+4.0	10.0	8.1
3	+7.0	8.0	9.5
4	+7.0	10.0	9.6
5	+9.5	8.0	10.6
6	+9.5	10.0	10.9
7	+14.0	8.0	12.6
8	+14.0	10.0	14.0

Table 5
Summary of Test Conditions; Plans 4R and 6
Erosion Depth = -8.6 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	7.3
2	+4.0	8.0	8.1
3	+4.0	10.0	7.6
4	+4.0	10.0	8.1
5	+7.0	8.0	8.5
6	+7.0	8.0	9.5
7	+7.0	10.0	8.6
8	+7.0	10.0	9.6
9	+9.5	8.0	9.6
10	+9.5	8.0	10.6
11	+9.5	10.0	10.1
12	+9.5	10.0	10.9
13	+14.0	8.0	11.0
14	+14.0	8.0	12.6
15	+14.0	10.0	12.5
16	+14.0	10.0	14.0

Table 6
Summary of Test Conditions; Plan 5R
Erosion Depth = -3.6 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	5.8
2	+4.0	8.0	6.3
3	+4.0	10.0	6.3
4	+4.0	10.0	7.0
5	+7.0	8.0	8.1
6	+7.0	8.0	8.9
7	+7.0	10.0	8.7
8	+7.0	10.0	9.6
9	+9.5	8.0	9.3
10	+9.5	8.0	10.1
11	+9.5	10.0	9.4
12	+9.5	10.0	10.7
13	+14.0	8.0	10.8
14	+14.0	8.0	12.2
15	+14.0	10.0	11.9
16	+14.0	10.0	13.2

Table 7
Summary of Test Conditions; Plans 7, 8, and 9
Erosion Depth = -10.0 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	7.8
2	+4.0	8.0	9.9
3	+4.0	10.0	8.0
4	+4.0	10.0	10.4
5	+7.0	8.0	9.0
6	+7.0	8.0	11.6
7	+7.0	10.0	9.5
8	+7.0	10.0	12.3
9	+9.5	8.0	10.3
10	+9.5	8.0	12.8
11	+9.5	10.0	10.9
12	+9.5	10.0	13.8
13	+11.5	8.0	10.6
14	+11.5	8.0	13.3
15	+11.5	10.0	11.5
16	+11.5	10.0	14.5
17	+14.0	8.0	11.4
18	+14.0	8.0	14.4
19	+14.0	10.0	12.1
20	+14.0	10.0	15.6

Table 8
Summary of Test Conditions; Plans 7R, 8R, and 9R
Erosion Depth = -10.0 ft mlt

Step	SWL ft, mlt	Wave Period T_p, sec	Wave Height H_{mo}, ft
1	+4.0	8.0	9.9
2	+4.0	10.0	10.4
3	+7.0	8.0	11.6
4	+7.0	10.0	12.3
5	+9.5	8.0	12.8
6	+9.5	10.0	13.8
7	+11.5	8.0	13.3
8	+11.5	10.0	14.5
9	+14.0	8.0	14.4
10	+14.0	10.0	15.6

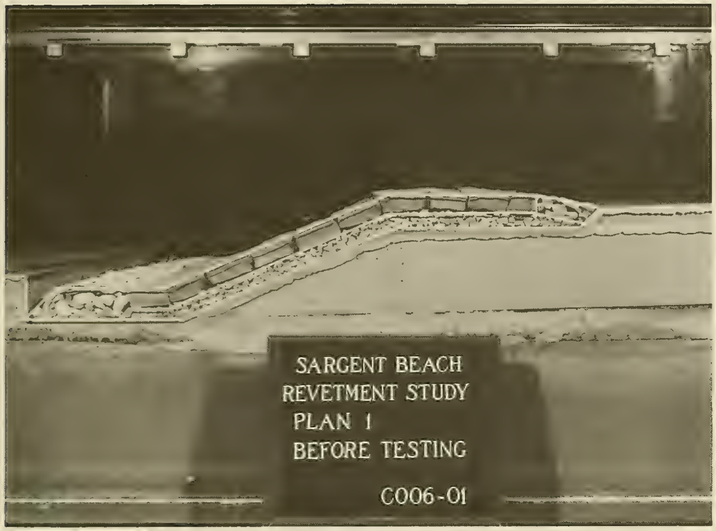

Photo 1. Side view of Plan 1 before wave attack

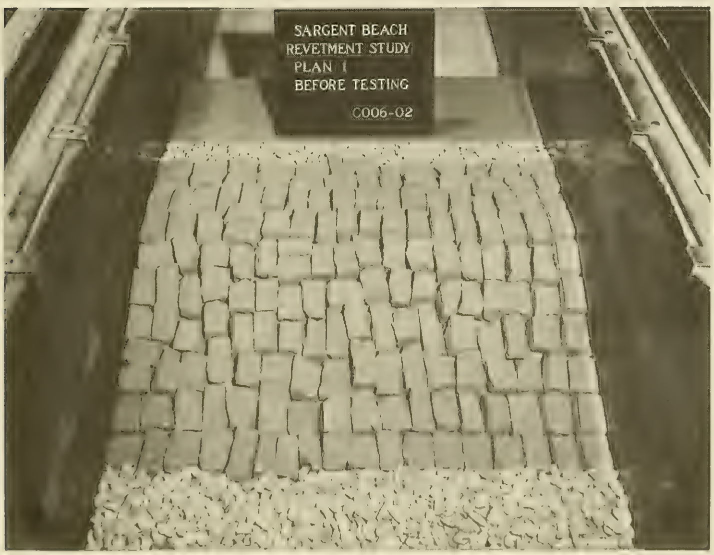

Photo 2. Sea-side view of Plan 1 before wave attack

Photo 3. Side view of Plan 1 after wave attack

Photo 4. Sea-side view of Plan 1 after wave attack

Photo 5. Land-side view of Plan 1 after wave attack

Photo 6. Side view of Plan 2 after wave attack

Photo 7. Sea-side view of Plan 2 after wave attack

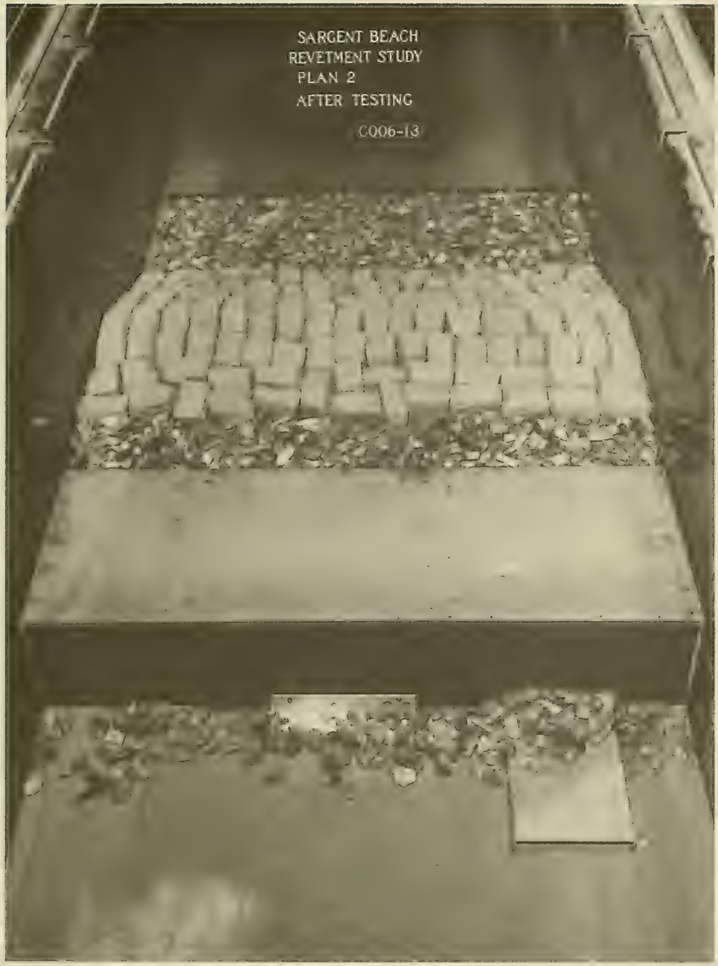

Photo 8. Land-side view of Plan 2 after wave attack

Photo 9. Side view of Plan 3 after wave attack

Photo 10. Sea-side view of Plan 3 after wave attack

Photo 11. Side view of Plan 3R after wave attack

Photo 12. Sea-side view of Plan 3R after wave attack

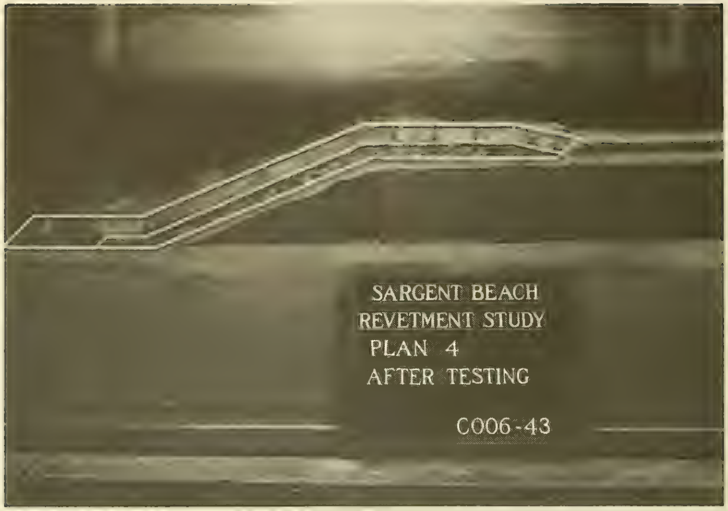

Photo 13. Side view of Plan 4 after wave attack

Photo 14. Sea-side view of Plan 4 after wave attack

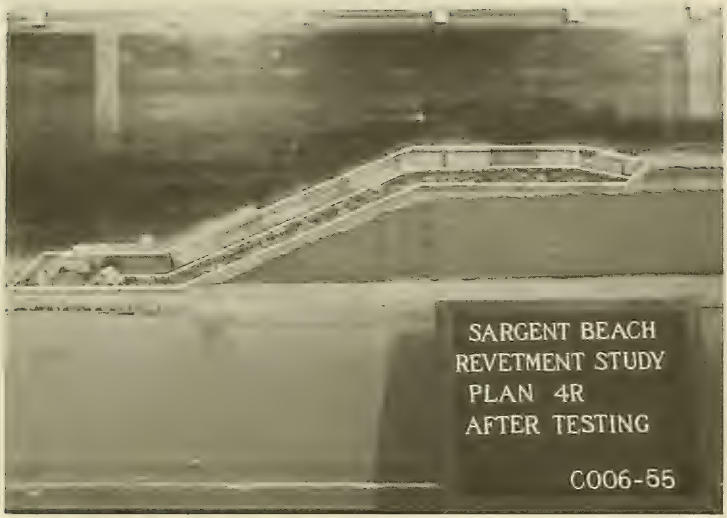

Photo 15. Side view of Plan 4R after wave attack

Photo 16. Sea-side view of Plan 4R after wave attack

Photo 17. Side view of Plan 5 after wave attack

Photo 18. Sea-side view of Plan 5 after wave attack

Photo 19. Side view of Plan 5R after wave attack

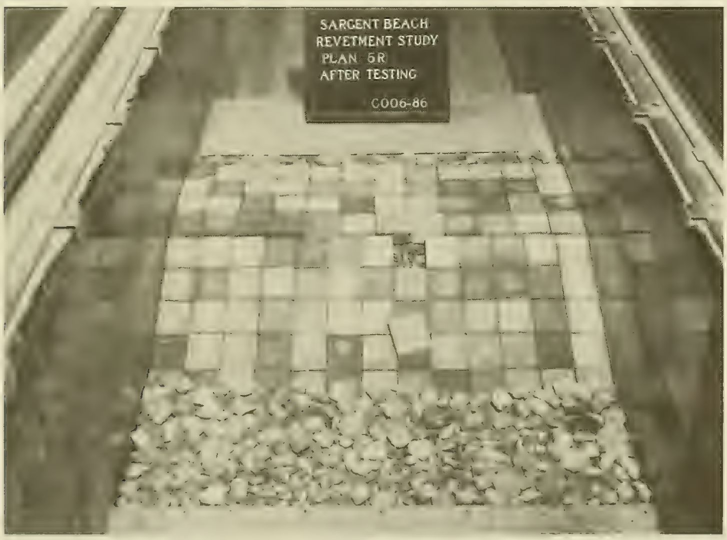

Photo 20. Sea-side view of Plan 5R after wave attack

Photo 21. Land-side view of Plan 5R after wave attack

Photo 22. Side view of Plan 6 before wave attack

Photo 23. Sea-side view of Plan 6 before wave attack

Photo 24. Land-side view of Plan 6 before wave attack

Photo 25. Side view of Plan 6 after testing

Photo 26. Sea-side view of Plan 6 after wave attack

Photo 27. Land-side view of Plan 6 after wave attack

Photo 28. Side view of Plan 6R after testing

Photo 29. Sea-side view of Plan 6R after wave attack

Photo 30. Land-side view of Plan 6R after wave attack

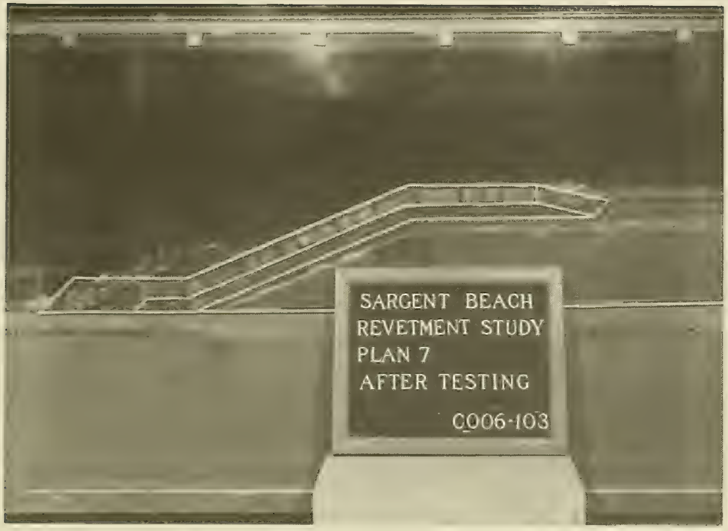

Photo 31. Side view of Plan 7 after wave attack

Photo 32. Sea-side view of Plan 7 after wave attack

Photo 33. Land-side view of Plan 7 after wave attack

Photo 34. Overhead view of Plan 7 after wave attack

Photo 35. Side view of Plan 7R after wave attack

Photo 36. Sea-side view of Plan 7R after wave attack

Photo 37. Land-side view of Plan 7R after wave attack

Photo 38. Overhead view of Plan 7R after wave attack

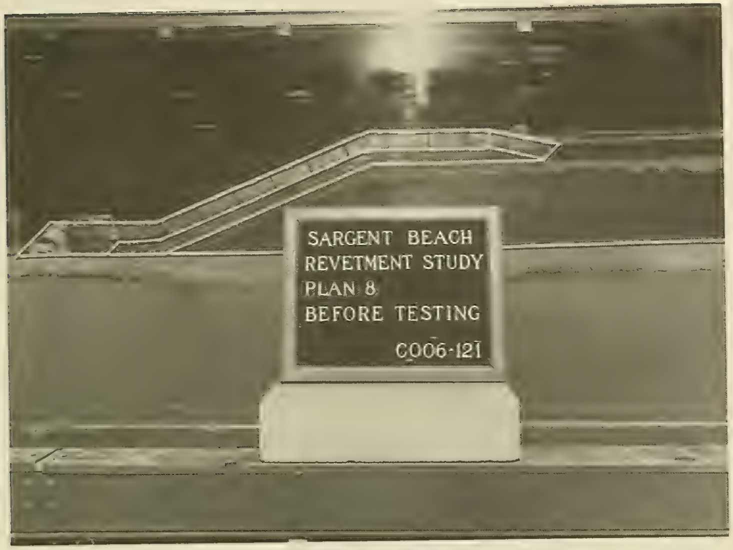

Photo 39. Side view of Plan 8 before wave attack

Photo 40. Sea-side view of Plan 8 before wave attack

Photo 41. Side view of Plan 8 after wave attack

Photo 42. Sea-side view of Plan 8 after wave attack

Photo 43. Land-side view of Plan 8 after wave attack

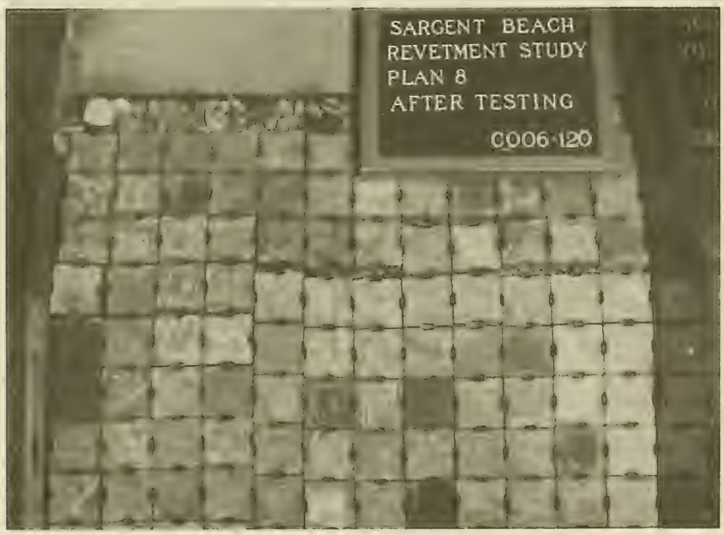

Photo 44. Overhead view of Plan 8 after wave attack

Photo 45. Sea-side view of Plan 8R after wave attack

Photo 46. Overhead view of Plan 8R after wave attack

Photo 47. Side view of Plan 9 after wave attack

Photo 48. Sea-side view of Plan 9 after wave attack

Photo 49. Overhead view of Plan 9 after wave attack

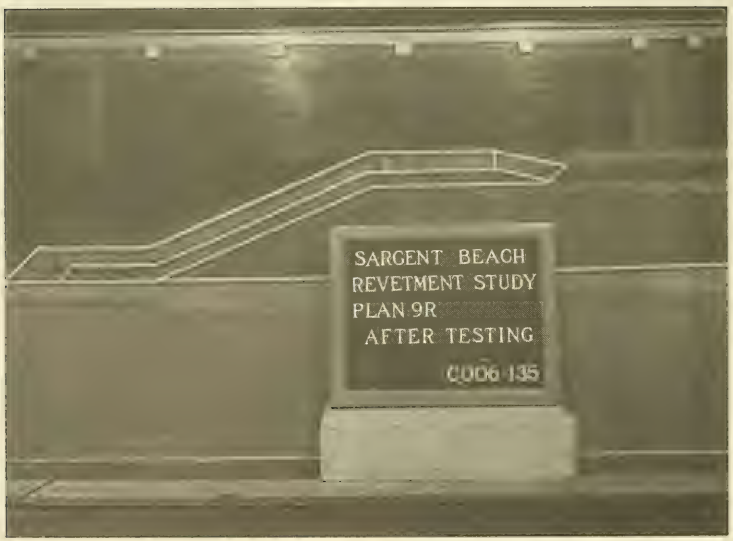

Photo 50. Side view of Plan 9R after wave attack

Photo 51. Sea-side view of Plan 9R after wave attack

Photo 52. Overhead view of Plan 9R after wave attack

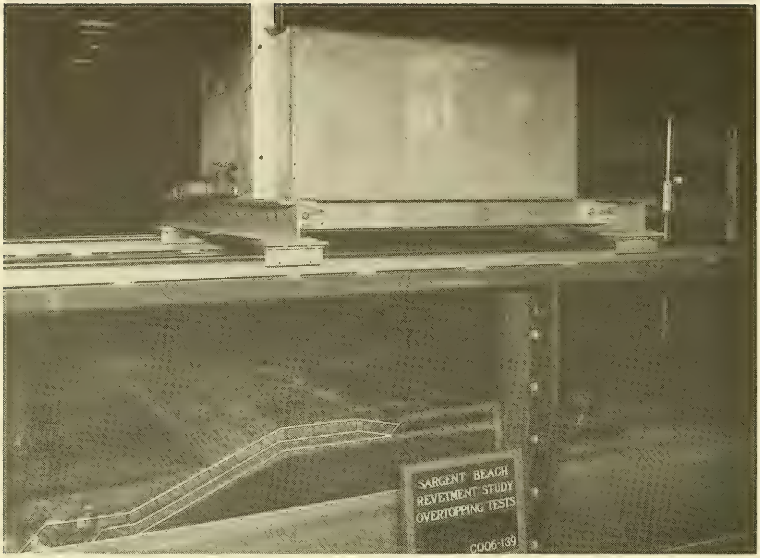

Photo 53. Setup for wave overtopping tests

Appendix A
Notation

H_{mo}	Zero-moment wave height, ft
L	Length
L^2	Area
L^3	Volume
L_m/L_p	Linear scale of the model
m	Model quantity
P	Prototype quantity
Q	Overtopping rate, cfs/ft
S_a	Specific gravity of an individual armor unit relative to the water in which it was placed
T	Time
T_p	Wave period of peak energy density of spectrum, sec
W_a	Weight of individual armor unit, lb

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13. ABSTRACT *(Maximum 200 words)*

The objective of this study was to investigate, via a two-dimensional (2-D) coastal model, alternate designs for the proposed revetment. Tests were conducted at a geometrically undistorted scale of 1:24, model to prototype.

Based on test results, it was concluded that:

a. Four- to six-ton armor stone is stable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at still-water levels (swl's) of +4.0 to +14.0 ft mean low tide (mlt) with assumed scour depths of -3.6 and -8.6 ft mlt.

b. Stability of the original concrete blocks, which were 6.0 ft by 5.5 ft by 2.5 ft and had 0 percent porosity, was only marginally acceptable for the maximum wave heights that can be expected to occur for 8- and 10-sec waves at swl's of +4.0 to +14.0 ft mlt with an assumed scour depth of -3.6 ft mlt.

c. Several modified block plans were tested and it was determined that the optimum block size was 5.75 ft by 5.75 ft by 2.5 ft. These blocks, weighing 6 tons and having a porosity of 4 percent, should prove stable for the maximum wave heights that can be expected to occur for 8- to 10-sec waves at swl's of +4.0 to +14.0 ft mlt with an assumed scour depth of -10 ft mlt.

14. SUBJECT TERMS

Concrete blocks	Sargent Beach, Texas
Revetment	Stone armor

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